ACEE Int. J. on Civil and Environmental Engineering, Vol. 2, No. 1, Aug 2013

Seismic Vulnerability Assessment of Steel Moment Resisting Frame due to Infill Masonry Walls, Variation in Column Size and Horizontal Buckling Restrained Braces

MagarPatilH.R.[1] ,Jangid R.S.[2]
Department of Civil Engineering, IIT Bombay, Powai, Mumbai – 400 076.
[1] Ph.D. Student (e-mail: hemant.359267@gmail.com)
[2] Professor (e-mail: rsjangid@civil.iitb.ac.in)

Abstract: **Steel moment resisting frame with open first storey (soft storey) is known to perform well compared with the RC frames during strong earthquake shaking. The presence of masonry infill wall influences the overall behavior of the structure when subjected to lateral forces, when masonry infill are considered to interact with their surrounding frames the lateral stiffness and lateral load carrying capacity of structure largely increase. In this paper, the seismic vulnerability of building with soft storey is shown with an example of G+10 three dimensional (3D) steel frame. The open first storey is an important functional requirement of almost all the urban multi-storey buildings, and hence, cannot be eliminated. Hence some special measures need to be adopted for this specific situation. The under-lying principle of any solution to this problem is in increasing the stiffness's of the first storey such that the first storey stiffness is at least 50% as stiff as the second storey, i.e., soft first storeys are to be avoided, and providing adequate lateral strength in the first storey. In this paper, stiffness balancing is proposed between the first and second storey of a steel moment resisting frame building with open first storey and brick infills as described in models. A simple example building is analyzed by modeling it with nine different methods. The stiffness effect on the first storey is demonstrated through the lateral displacement profile of the building.**

Keywords: **Soft Storey, Infill walls, Steel Moment Resisting Frame, Earthquake.**

I. Introduction

Steel Moment Resisting Frames are more widely used in construction of industrial sheds. The plate girders, gantry girders, columns with gusseted base or slab base, trusses, purlins are some of the structural components in the industrial sheds. Masonry wall is very rarely used in industrial shed. Galvanized Iron sheet is particularly used instead of masonry wall in the industrial shed for partition purpose. But, for the delegate instruments, controlled room temperature is one of the basic needs hence masonry walls are provided in the industrial sheds.

Even though, RCC framed structures are more durable and have less maintenance, it has disadvantage like less ductility in lateral direction. It has been found that reinforced cement concrete columns and beams fails in bending and torsion which causes to expose steel during earthquake. After 1994, Northridge earthquake, people started searching an option which will have good ductility in horizontal direction. Hence, Steel Moment Resisting Frames (SMRF) with masonry infills became a popular form of construction of high-rise buildings in urban area around the world. The term infilled frame is used to denote a composite structure formed by the combination of a moment resisting plane frame and infill walls. The masonry can be of brick, concrete units, or stones. Usually the frame is filled with bricks as non-structural wall for partition of rooms. Social and functional needs for vehicle parking, shops, reception etc. are compelling to provide an open first storey in high rise building. Parking floor has become an unavoidable feature for the most of urban multistoried buildings. Though multistoried buildings with parking floor (soft storey) are vulnerable to collapse due to earthquake loads, their construction is still widespread. These buildings are generally designed as framed structures without regard to structural action of masonry infill walls. They are considered as non-structural elements. Due to this, in seismic action, steel frames purely acts as moment resisting frames leading to variation in expected structural response. The effect of infill panels on the response of steel frames subjected to seismic action is well recognized and has been subject of numerous experimental and analytical investigations over last six to seven decades. In the current practice of structural design in India, masonry infill panels are treated as non-structural elements and their strength and stiffness contributions are neglected. In reality, the presence of infill wall changes the behavior of frame action into truss action thus changing the lateral load transfer mechanism. From the literature, it can be observed that current practice of multistoried building with soft first storey for earthquake forces are without consideration in reduction of stiffness at first floor. The sudden reduction in stiffness at first floor is most important part of the building analysis because it affects the concentration of forces at the first floor. In the present work, analytical investigations are carried out to study the effectiveness of different stiffening measures to the first storey for better performance of the structure during earthquake [9]. Different types of analytical models based on the physical understanding of the overall behavior of an infill panels were developed

DOI: 01.IJCEE.2.1.1197

over the years to mimic the behavior of infilled frames. The available infill analytical models can be broadly categorized as Macro Model and Micro models. The single strut model is the most widely used as it is simple and evidently most suitable for large structures. Thus steel frames with unreinforced masonry walls can be modelled as equivalent braced frames with infill walls replaced by equivalent diagonal strut which can be used in rigorous nonlinear pushover analysis. The theory of beams on elastic foundations suggests a non-dimensional parameter to determine the width and relative stiffness of diagonal strut [5]. Developing effective seismic protective systems for new or existing structures requires striking a balance between stiffness, strength and damping [1,2,3,4,10,11]. The carefully detailed structures using seismic or base isolation and a structural control device to dissipate energy to reduce deformations are the primary options to successfully create the necessary balance [12,15,16,17,18]. The system designed according to a prescriptive approach, deforms inelastically, results in significant damage after an event. The damage caused by the 1994 Northridge, California and 1995 Kobe, Japan earthquakes initiated a re-examination of connection details. Hence, to improve structural performance and predictable damage during seismic events, provide a seismic protective system within the framework of performance-based design [6,7,8,13,14].

II. Soft Storey

Construction of multistoried steel frame buildings is very rare in India. The most common type of vertical irregularity occurs in buildings that have an open ground storey. Many buildings constructed in recent time have a special feature that the ground stories are left open for the purpose of parking, reception etc. Such buildings are often called open ground storey buildings or buildings on stilts. The first stories become soft and weak relative to the other upper stories, due to absence of masonry walls in the first storey. Structurally those unbalances are unhealthy and soft storey buildings are well known for being susceptible to collapse through past earthquakes. A soft storey is one in which the lateral stiffness is less than 70 percent of that in the storey above or less than 80 percent of average lateral stiffness of the three stories above.

A. Behavior of Soft Storey

Experience in the past earthquake has shown that the buildings with simple and uniform configurations are subjected to less damage. Regularity and continuity of stiffness in the horizontal planes as well as in vertical direction is very important from earthquake safety point of view. A building with discontinuity is subjected to concentration of forces and deformations at the point of discontinuity which may leads to the failure of members at the junction and collapse of building. The total seismic base shear, experienced by a building during earthquake depends on its natural period. The seismic force distribution is dependent on the distribution of stiffness and mass along the height. The essential characteristics of soft storey consist of discontinuity of strength or stiffness which occurs at the second floor column connection. This discontinuity is caused because of lesser strength or increased flexibility in the first floor vertical structure results in extreme deflection in the first floor. If all the floors are approximately equal in strength and stiffness, the entire building deflection under earthquake load is distributed approximately equally at each floor (Figure1).

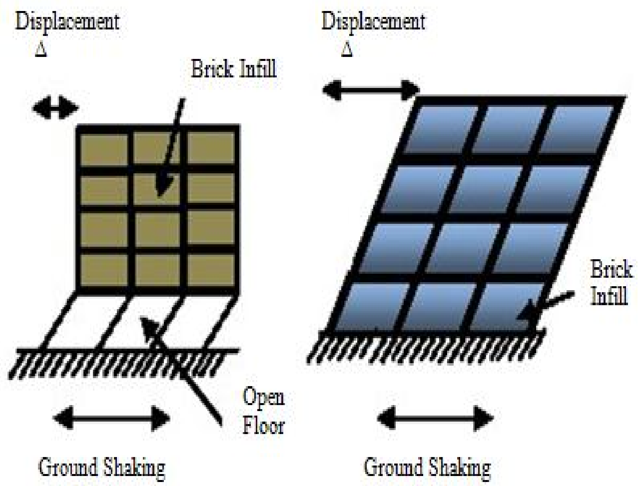

Figure 1 Soft First Storey

In buildings with soft first storey, the inter-storey drift in the soft first storey is very large which results in the increased strength demand on the column in the first storey for these building. However due to presence of brick infill walls in the upper stories, the forces in the columns are effectively reduces as walls shares some of the forces. If the first floor is significantly less strong or more flexible, a large portion of the total building deflections tends to concentrate in that floor. The presence of walls in upper stories makes them much stiffer than the open ground storey. Thus the upper stories move almost together as a single block and most of the horizontal displacement of the building occurs in the soft ground storey. Thus, such building behave like an inverted pendulum with the ground storey columns acting as the pendulum rod and the rest of the building acting as a rigid pendulum mass during earthquake as shown in the figure 2. As a consequence, large movement occurs in the ground storey alone and the columns in the open ground storey are severely stressed. If the columns are weak (do not have the required strength to resist these high stresses), they may be severely damaged which may even lead to collapse of the building.

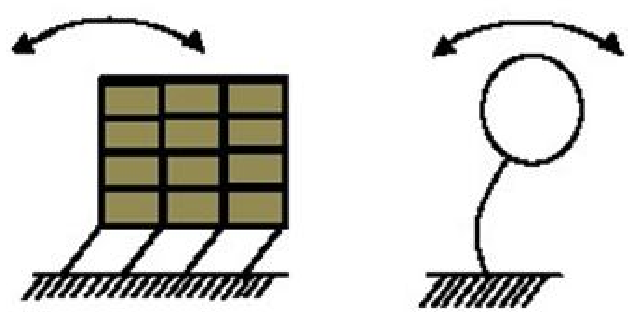

Figure 2 Behavior of Frames under Lateral Loads

DOI: 01.IJCEE.2.1.1197

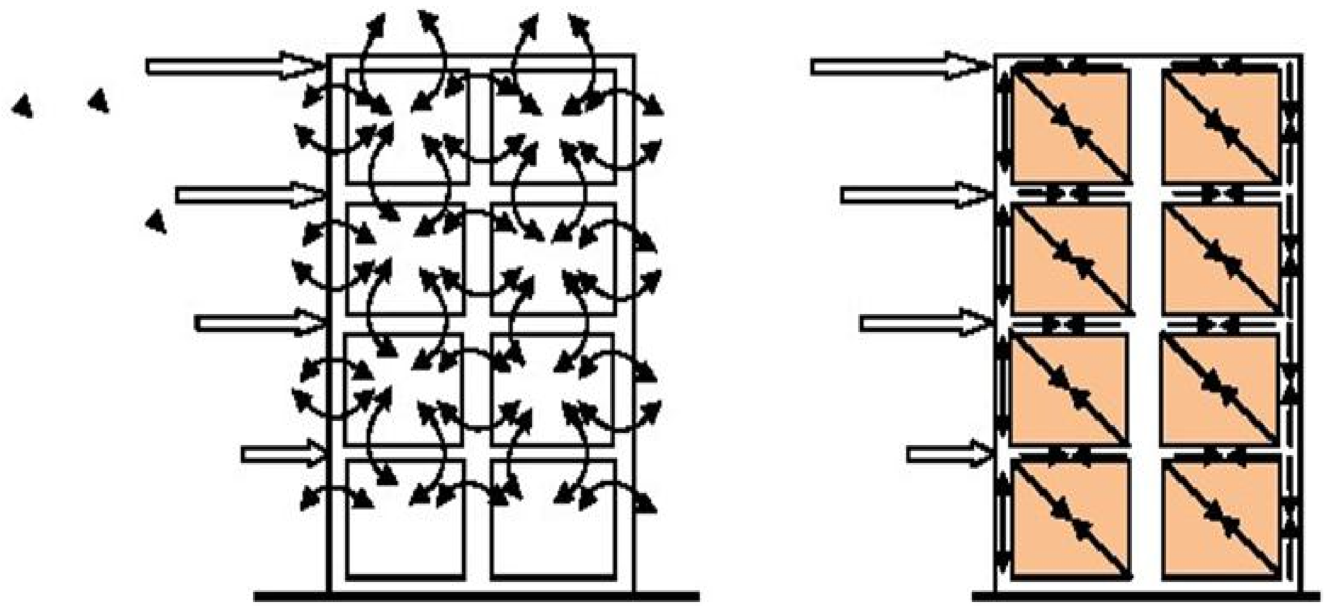

Figure 3 Change in lateral-load transfer mechanism due to masonry infill

In general practice, infill panels are assumed as non- structural elements and their strengths and stiffnesses are neglected in analysis and design of structures, while their mass is taken into account for load calculations. Infills alter the behavior of building from frame action to truss action and carry the lateral seismic force as a compressive axial force along their diagonals as shown in Figure 3. Infill walls act as diagonal struts and increase the stiffness of framed buildings. The increase in the stiffness depends on the wall thickness and the number of frame panels with infills, and can be quite significant in some cases. The increased stiffness of the building due to the presence of infills reduces the ability of the frame to flex and deform. In steel moments resisting frames, masonry infills may prevent the primary frame elements (i.e., columns and beams) from responding in a ductile manner — instead, such structures may show a non ductile (brittle) performance. This may culminate in a sudden and dramatic failure.

The focus of this report is on seismic vulnerability assessment of steel moment resisting frame due to masonry infill walls, variation in column cross sections and also due to use of steel horizontal strut. Behavior of structures varies with and without use of masonry infill walls. Also, Variation in cross section of columns and use of horizontal mild steel strut will play vital role in improvement of stiffness of columns. Nine different seismic resisting systems are studied and compared. Several damage measures such as maximum roof displacement, base shear, and total roof and storey acceleration are studied.

Behavior of structure subjected to earthquake loading is a complicated phenomenon. For this study, various building models with different improving measures are considered as described as above. The research is mainly focused on the behavior of steel moment resisting frame with soft first storey. Masonry infill panels are modeled as equivalent diagonal strut in the upper stories as shown in Figure 6 [5]. Three dimensional analysis of building is carried out for earthquake zone III. The linear static analysis is carried out on all mathematical 3D models using the finite element method based software SAP 2000. The results obtained from the analysis are discussed in next sections.

Linear static analysis is performed on nine models of the building using SAP 2000 analysis package. The frame members are modelled with rigid ends, the walls are modelled as diagonal struts and the floors are modelled as diaphragms rigid in-plane. Time history analysis is performed on the nine models of the building considered in this study with the time history plot of El-Centro and Northridge earthquake.

Linear Static Analysis of steel frame building will be done to analyse following models (Figure 4 and 5) using SAP 2000:

M1 → Bare Frame
M2 → Frame with first soft storey
M3 → Frame with walls in all stories and also at some selected positions in first storey
M4 → Frame with walls in all stories and also at some selected positions in first storey along with stiffer columns
M5 → Frame with walls in all stories along with stiffer columns in first storey
M6 → Frame with first soft storey and columns with varying cross sections
M 7 → Frame with walls in all stories and columns with varying cross sections
M 8 →Frame with walls in all stories and also at some selected positions in first storey along with stiffer columns and a horizontal beam just below main beam
M 9 →Frame with first soft storey and columns with varying cross sections and a horizontal beam just below main beam

DOI: 01.IJCEE.2.1.1197

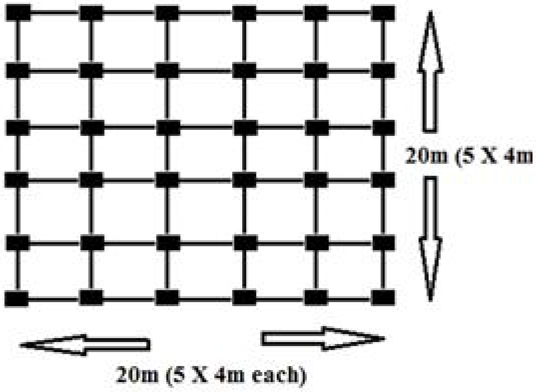

Figure 4 Plan for all Models

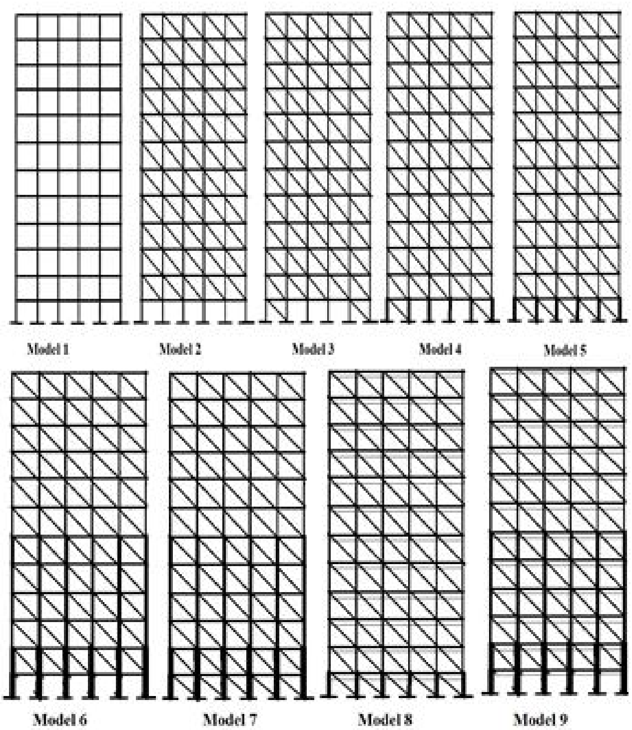

Figure 5 Elevation of all models

B. Storey Stiffness

The storey stiffness is defined as the magnitude of force couple required at the floor levels adjoining the storey to produce a unit lateral translation within the storey, letting all the other floors to move freely [5].

Stiffness of one column is equal to

$$Kc = 12EcI/ L^3$$

Stiffness of diagonal strut is equal to

$$Ki = (A\ Em / L_d) \cos^2\theta$$

Therefore total stiffness of one storey is

$$K = \sum Kc + \sum Ki$$

Where, K= Total Stiffness

Kc= Stiffness of Column

Ki= Stiffness of wall strut

Ec= Modulus of Elasticity of frame material

Em= Modulus of Elasticity of infill material

A= Area of diagonal strut

Ld= Effective length of diagonal strut

= Inclination of diagonal strut

ISMB 300,450,500 and 600 are used for beams and columns with combinations and M20 concrete has been used for slabs. The load combination has been used as per IS: 1893-2002.

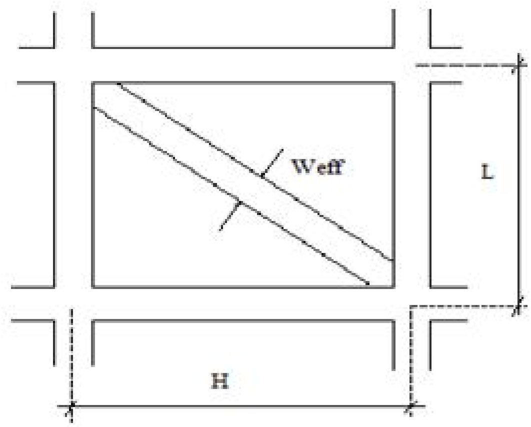

Figure 6 Compression Diagonal Strut Model

It is observed that the stiffness of first storey for model I is same as that of second storey stiffness in longitudinal as well as in transverse direction. The stiffness of first storey for model II is about 20.86% of second storey stiffness in longitudinal direction. Model II represent the realistic situation for earthquake. It is seen that use of brick infill at specific locations (Model III) reduces the stiffness irregularity marginally. In case of model III stiffness of first storey is increased to 42.28% of second storey stiffness. The use of stiffer columns in first storey (Model IV to IX) increases the stiffness in longitudinal as well as transverse direction.

III. Results and Discussion

A. Base Shear

Base shear is the total horizontal seismic shear force at the base of structure. The results obtained from Linear Static Analysis (LSA) regarding base shear along longitudinal direction for Model I to Model IX are presented in figures 7 to fig. 8 for Imperial Valley Time history and Northridge Time History. In case of both time histories, Models II to IX shows base shear values are nearly same, while, Model I having less value of base shear in X direction. Where as in Y direction, base shear values for Models III, IV, V and VII, VIII are nearly same while Models I, II, VI and IX have these values comparatively very less. This is because of Model I having large value of fundamental natural time period as compared to other models. Due to absence of masonry walls in Model I, fundamental natural period get increased and therefore base shear get reduced. Base shear response of models more in magnitude when diagonal wall strut comes into action. For models V, VII and VIII, response is comparatively less.

Base Shear in Columns: The maximum shear forces in X and Y direction in the columns are shown in figures 7 to 10; the shear force (strength) demands are severely higher for first

DOI: 01.IJCEE.2.1.1197

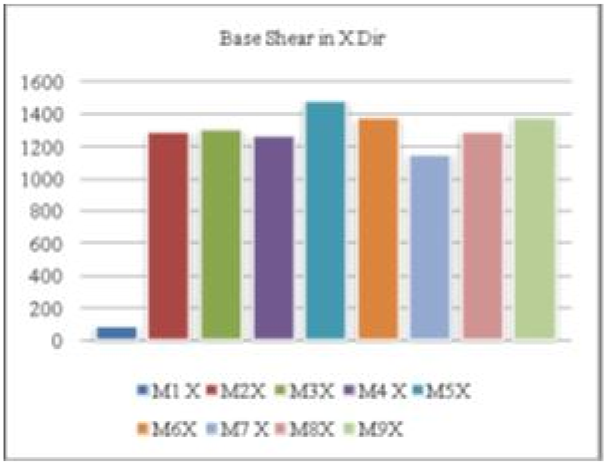

Figure 7 Comparative study of Base Shear in X direction for Imperial Valley time history

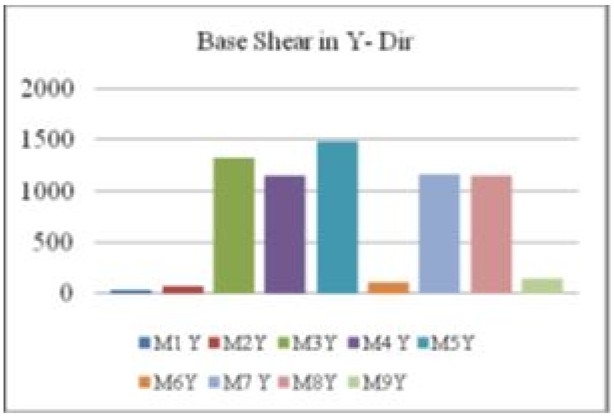

Figure 8 Comparative study of Base Shear in Y direction for Imperial Valley time history

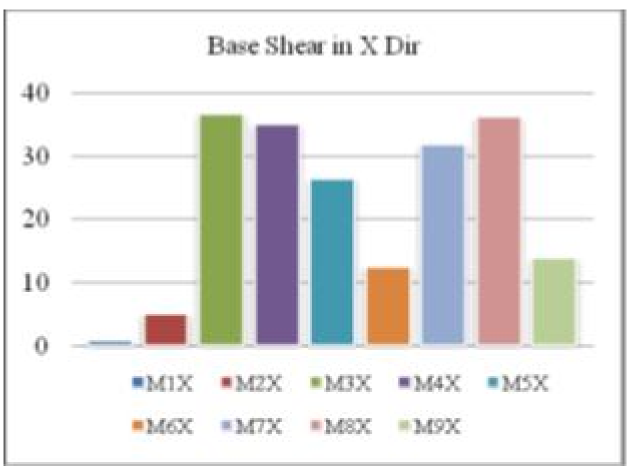

Figure 9 Comparative study of Base Shear in X Direction for Northridge time history

storey columns, in case of the soft first storey buildings. The force is distributed in proportion to the stiffness of the members, the force in the columns of the upper storeys, for all the models (except model I), are significantly reduced. When the bare frame model is subjected to earthquake load, mass of each floor acts independently which causes each floor to drift with respect to adjacent floors. Thus the building frame behaves in a flexible manner causing distribution of horizon

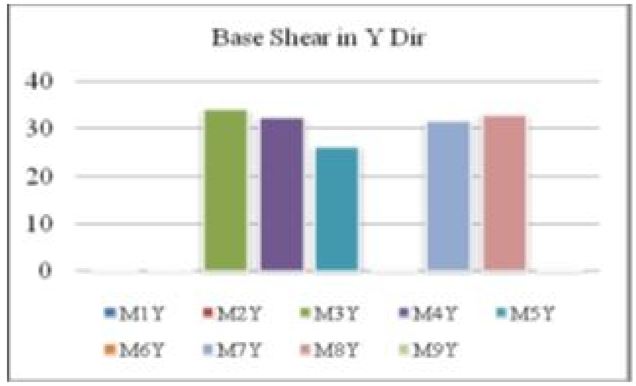

Fig. 10 Comparative study of Base Shear in Y Direction for Northridge time history

tal shear across floors. In presence of infill, the relative drift between adjacent floors is restricted causing mass of the upper floors to act together as a single mass. In such a case, the total inertia of the all upper floors causes a significant increase in the horizontal shear at base or in the ground floor columns. It is observed that for Model I base shear value is less and displacement is more as compared to other models. Other models having nearly same base shear value but there is difference in displacement.

B. Displacement

The displacement of Model I i.e. bare frame is 6.95 mm, which is more than the other models. Model II to IX having displacement values 6.6mm, 4.21mm, 3.68mm, 3.35mm, 4.38mm, 3.15mm, 3.73mm and 4.22mm respectively. Displacement profile of the building can be seen in figures 11 to 14. This indicates the influence of moment of inertia and linear as well as lateral stiffness on the displacement of the structure. The reduction in displacement is attributed due to the enhanced stiffness of the structure. For models II, VI and IX displacement is suddenly increasing by large magnitude at first soft storey where as for all other models, it is increasing linearly.

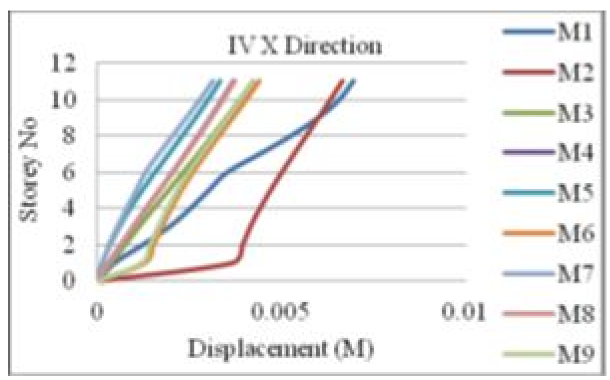

Figure 11 Displacement of Column 01 in X Direction for Imperial Valley time history

The lateral displacement profiles of the various models for the two different analysis performed in this study are shown in figures 3.5 to 3.8 in X and Y directions for Imperial valley and Northridge time histories respectively. In these figures, the abrupt changes in the slope of the profile indicate the stiffness irregularity. All displacement profiles

DOI: 01.IJCEE.2.1.1197

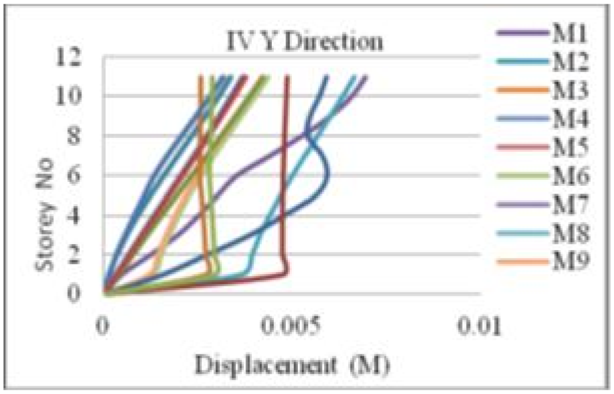

Figure 12 Displacement of Column 01 in Y Direction for Imperial Valley time history

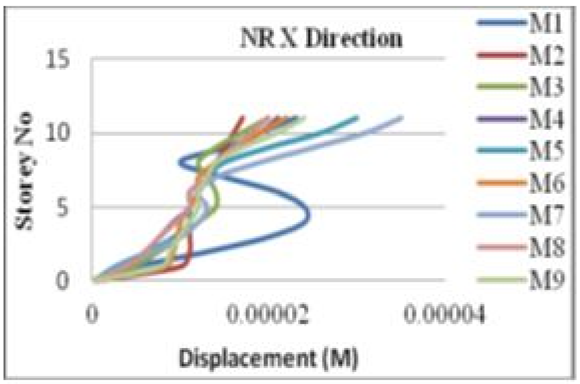

Figure 13 Displacement of Column 01 in X direction for Northridge time history

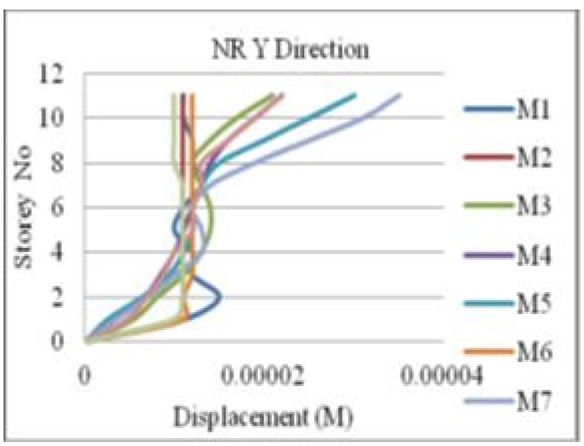

Figure 14 Displacement of Column 01 in Y Direction for Northridge time history

corresponding to model I having stiffness irregularity have a sudden change of slope at first floor level.

However, the other models i.e. II, III & VI, shows smooth displacement profiles. The inter-storey drift demand is largest in the first storey for all the models with soft ground storey. This implies that the ductility demand on the columns in the first storey, for these models, is the largest.

C. Roof Acceleration

Accelerations of models other than model one is comparatively more. For model five, magnitude of acceleration is more than any other model. Figures 15 to 18 show the comparison of roof acceleration in X and Y directions for Imperial Valley and Northridge earthquake time history respectively. If this acceleration is viewed with respect to base shear, it is found that acceleration varies in proportion with base shear. For model number five, base shear is also very high. Figure 19 to figure 22 shows that Roof acceleration response due to Imperial Valley and Northridge earthquake time history indicates that response is more when diagonal wall strut plays their role and it decreases its magnitude when diagonal wall struts are not in tension.

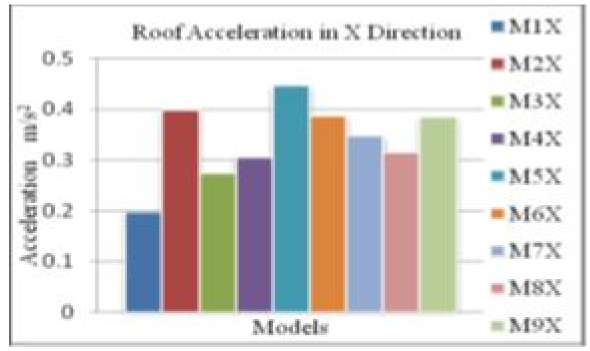

Figure 15 Roof Acceleration analysis in X for Imperial Valley time history

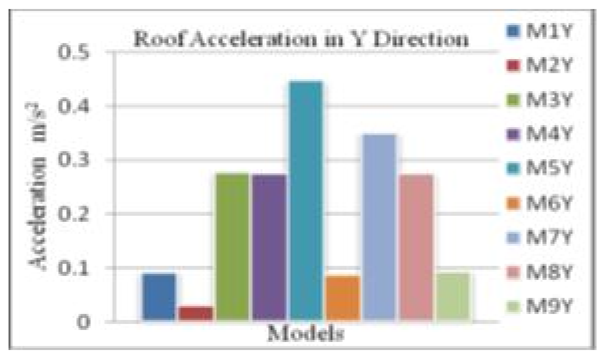

Figure 16 Roof Acceleration analysis in Y for Imperial Valley time history

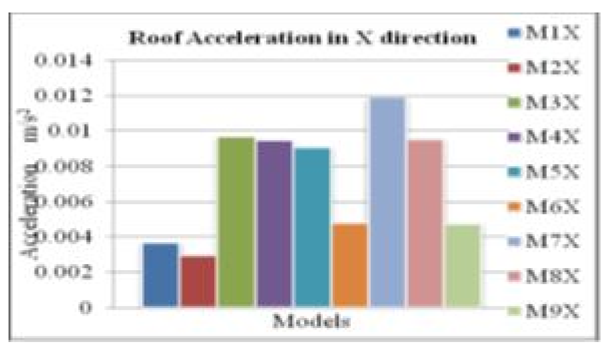

Figure 17 Roof Displacement analysis in X for Northridge time history

IV. Conclusions

In this paper, the seismic vulnerability of steel moment resisting frame buildings with and without soft first storey is shown through examples of building. The drift and the strength demands in the first storey columns are very large for buildings with soft ground storeys. It is not very easy to

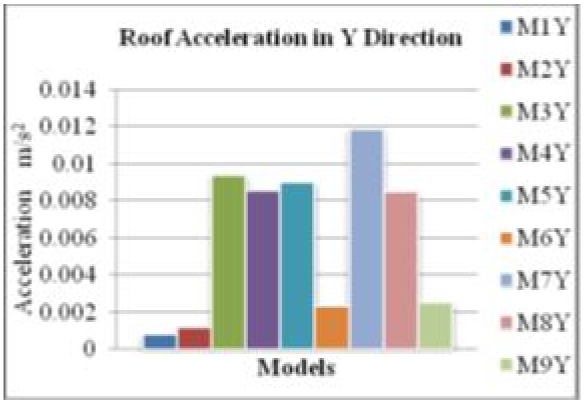

Figure 18 Roof Displacement analysis in Y for Northridge time history

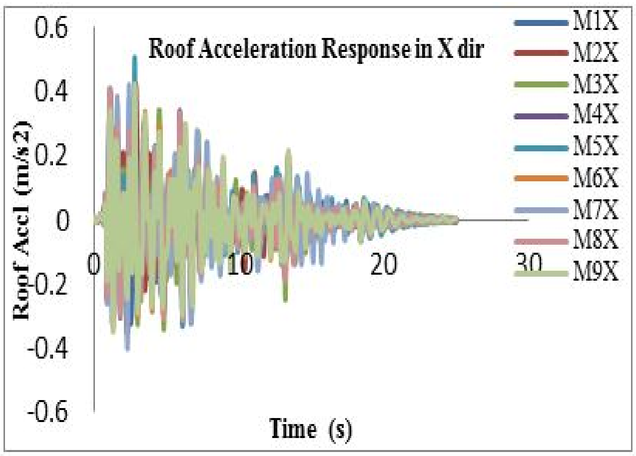

Figure 19 Roof Acceleration Response in X direction for Imperial Valley time history

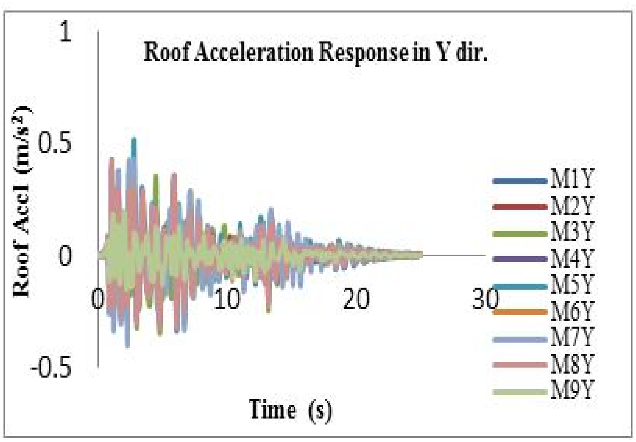

Figure 20 Roof Acceleration Response in Y direction for Imperial Valley time history

provide such capacities in the columns of the first storey. Thus, it is clear that such buildings will exhibit poor performance during a strong shaking. This hazardous feature of steel frame buildings needs to be recognized immediately and necessary measures needs to be taken to improve the performance of the buildings. The open first storey is an important functional requirement of almost all the urban multi-storey buildings, and hence, cannot be eliminated. Alternative measures need to be adopted for this specific situation. The under-lying principle of any solution to this problem is in

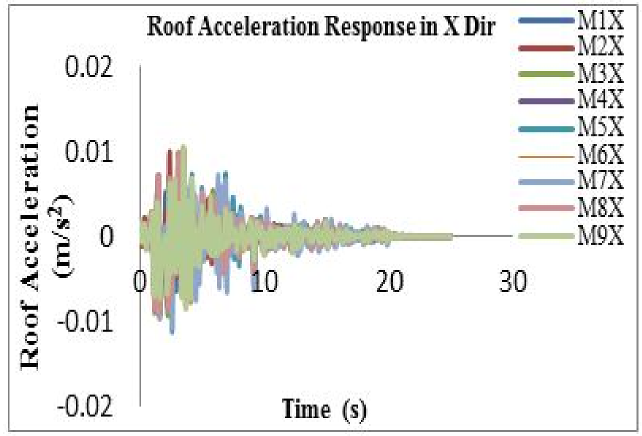

Figure 21 Roof Acceleration Response in X direction for Northridge time history

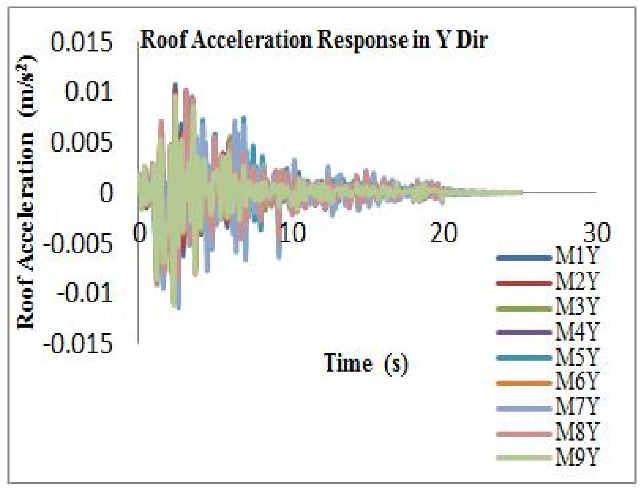

Figure 22 Roof Acceleration Response in Y direction for Northridge time history

increasing the stiffnesses of the first storey such that the first storey stiffness is at least 50% as stiff as the second storey, i.e., soft first storeys are to be avoided, and providing adequate lateral strength in the first storey. The possible schemes to achieve the above are provision of stiffer columns in the first storey, and provision of infill wall at specified location at ground floor in the building. The former is effective only in reducing the lateral drift demand on the first storey columns. However, the latter is effective in reducing the drift as well as the strength demands on the first storey columns. Hence, model IV i.e. soft first storey with walls at specific locations in first storey along with stiffer columns is proposed from this research to resist displacement due to time history analysis.

References

[1] Amadio C, Clemente I, Macorini L, Fragiacomo M. Seismic Behaviour of Hybrid Systems Made of PR Composite Frames Coupled with Dissipative Bracings. Earthquake Engineering and Structural Dynamics 2008; 37(6):861–879.

[2] Bartera F, Giacchetti R. Steel Dissipating Braces for Upgrading Existing Building Frames. Journal of Constructional Steel Research 2004; 60(3–5):751–769.

DOI: 01.IJCEE.2.1.1197

[3] Black CJ, Makris N, Aiken ID. Component Testing, Seismic Evaluation and Characterization of Buckling Restrained Braces. Journal of Structural Engineering 2004; 130(6):880–894.

[4] Constantinou MC, Tsopelas P, Hammel W, Sigaher AN. Toggle Brace Damper Seismic Energy Dissipation Systems. Journal of Structural Engineering 2001; 127(2):105–112.

[5] Das, D., Murty, C. V. R. (2004) „„Brick masonry infills in seismic design of RC framed buildings: Part 1 – Cost implications" The Indian Concrete Journal July 2004, vol78 No7: 39-43.

[6] Durgesh C. Rai, Benjamin J. Wallace, Aluminium Shear Link for Enhanced Seismic Resistance, Earthquake Engineering And Structural Dynamics Earthquake Engg. Struct.Dyn. 27, 315-342 (1998)

[7] D.R. Sahoo& D.C. Rai, Seismic Strengthening of RC Buildings using steel cage and Aluminium cage, 8B177_8PCEE_Rai,2007

[8] FEMA. State of the Art Report on Systems Performance of Steel Moment Frames Subject to Earthquake Ground Shaking (FEMA 355C). Federal Emergency Management Agency: Washington, D.C., 2000b.

[9] FEMA. Next Generation Performance Based Seismic Design Guidelines (FEMA 445). Federal Emergency Management Agency: Washington, D.C., 2006.

[10] J.D. Marshall, F.A. Charney, Seismic response of steel frame structures with hybrid passive control systems, Earthquake Engineering And Structural Dynamics Earthquake Engg Struct. Dyn. (2011), Published online in Wiley Online Library

[11] J. D. Marshall and F. A. Charney, Dynamic Response of Steel Moment-Frame Structures with Hybrid Passive Control Systems, ATC & SEI 2009 Conference on Improving the Seismic Performance of Existing Buildings and Other Structures 2009,1022-1033)

[12] Mazza F, Vulcano A. Control of the Earthquake and Wind Dynamic Response of Steel Framed Buildings by Using Additional Braces and/or Viscoelastic Dampers. Earthquake Engineering & Structural Dynamics 2011; 40 (2):155–174.

[13] M.C. Constantinou, M.D. Symans Experimental and Analytical Investigation of Seismic Response of Structures with Supplemental Fluid Viscous Dampers, NCEER, State University of New York at Buffalo, December 21, 1992.

[14] Naeim F, Kelly J. Design of Seismic Isolated Structures: From Theory to Practice. John Wiley and Sons, Inc.; New York, NY, 1999.

[15] Pall AS, Marsh C. Response of Friction Damped Braced Frames. Journal of the Structural Division, ASCE 1982; 108(ST6):1313–1323.

[16] Symans MD, Charney FA, Whittaker AS, Constantinou MC, Kircher CA, Johnson MW, McNamara RJ. Energy Dissipation Systems for Seismic Applications: Current Practice and Recent Developments. Journal of Structural Engineering 2008; 134(1):3–21.

[17] Tremblay R, Lacerte M, Christopoulos C. Seismic Response of Multistory Buildings with Self Centering Energy Dissipative Steel Braces. Journal of Structural Engineering 2008; 134(1):108–120.

[18] Ragni L, Dezi L, Dall'Asta A, Leoni G. HDR Devices for the Seismic Protection of Frame Structures: Experimental Results and Numerical Simulations. Earthquake Engineering and Structural Dynamics, 2009.

DOI: 01.IJCEE.2.1.1197